

Flexural Strengthening of Reinforced Concrete Girders using Post-Tensioned Concrete Jackets

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Abstract— This study discusses the flexural behavior of reinforced concrete girders strengthened using post-tensioning embedded in concrete jackets. The concept benefits from the external jacket to help increasing the cross-section inertia as well as to host the post-tensioning tendons without the need of external deviators. This results in a significant enhancement to the strength and stiffness of the original girder. The experimental phase of the study was conducted on two stages the first deals with girders loaded on their original section firstly and then strengthened with the jacket and loaded to failure, while the second had the girders that were strengthened before being subjected to loads. In addition to the stage of jacket introduction, the difference between the original girder and the jacket's concrete compressive strength was also studied. In the analytical phase of the study, a numerical model was built using the finite element method to simulate the response of the tested girders in the two experimental stages. This paper presents findings of the experimental program as well as the comparison with the analytical results of the model which showed a close correlation. This model may then be used with confidence to conduct an extensive analytical study for untested parameters.

Keywords— Flexural strengthening, reinforced concrete, post-tensioned concrete, jackets, finite element model, ABAQUS.

I. INTRODUCTION

Rehabilitation of existing structures has always been an important sector of structural engineering. Sometimes, repair of deteriorated concrete structures is more economical than building new one, if by repairing a safe and serviceable structure can be achieved. Strengthening for load capacity increase is also a crucial aspect of the rehabilitation industry. The success of a repair or a rehabilitation project will depend on the degree to which the work is executed in conformance with plans and specifications [1]. Strengthening of girders in flexure can be

achieved by adding a new structural element to the section, either steel or reinforced concrete [2] or most recently, Fiber Reinforced Polymers. The necessity to rehabilitate a Reinforced Concrete (RC) structure emerges from several reasons such as new safety requirements, change of structure occupancy, incorrect design calculations and/or degradation of materials with time. One of the most commonly used mitigation practices to strengthen and repair RC girders is the application of RC jackets to the girders sides [3].

Strengthening of concrete structures using external prestressing tendons has now been used for some time. External prestressing was initially developed for retrofitting of bridges, but now it is used for both retrofitting and in building new structures. Due to their simplicity and cost effectiveness, pre-stressed concrete bridges with external prestressing are becoming popular [4]. It has been found to provide an efficient and economical solution for a wide range of bridge types and conditions. The technique is growing in popularity because of the speed of installation and the minimal disruption to traffic flow. It increases the flexural and shear strength of strengthened girders as well as increasing stiffness which will reduce deflection. This technique is more economic compared to other methods [5].

II. PREVIOUS STUDIES ON STRENGTHENING RC GIRDERS

2.1 Previous Studies using the External Prestressing Technique

Jafar Sadak Ali et al, [6] introduced a method for the calculation of cable strain, which is based on the deformation compatibility of girder and friction at the deviators, was proposed to predict entire response of externally pre-stressed concrete girders up to elastic limit. Application of the developed method in numerical analysis on a rectangular girder with different profiles of prestressing cable was then performed. An algorithm has been developed to determine the structural behavior at the

deviator points in an externally pre-stressed girder. The predicted results showed that the structural behavior of externally pre-stressed concrete girders could be satisfactorily predicted from zero loading stage up to the proportional limit loading stage for different cable profiles.

Ali Hussein et al, [7], presented a nonlinear finite-element analysis to investigate the behavior up to failure of continuous composite steel-concrete girder with external prestressing tendons, in which a concrete slab is connected with steel I-girder by means of headed stud shear connectors, subjected to symmetrically static loading. ANSYS computer program (version 12.1) has been used to analyze the three-dimensional model. This covers: load deflection behavior, strain in concrete, and strain in steel girder and failure modes. The results obtained by finite element solutions have shown good agreement with experimental result.

M. A. Algorafi et al, [8], worked in an experimental investigation of the structural behavior of externally pre-stressed segmented (EPS) bridged under combined bending, shear, normal, and torsion stresses. A parametric study was carried out to investigate the effect of different external tendon layouts and different levels of torsion. The many advantages of this type of structure include offering fast and versatile construction, no disruption at ground level, high controlled quality and cost savings that have made them the preferred solution for many long-elevated highways.

Swoo-Heon Lee et al, [9], conducted a full-scale experimental study assessed the behavior of continuous concrete girders retrofitted with external pre-stressed bars. Three three-span girders were tested in two-point loading of the interior span. The results indicate that the external prestressing increased the load-carrying capacity by about 25% and the flexural stiffness by about 15%.

Mohamed H. Harajli, [10], tested sixteen girders, in which they were firstly subjected to cyclic fatigue loading at a constant load range to induce fatigue deformations. Then, they were externally pre-stressed and subjected to monotonically increasing load to failure. The nominal flexural strengths of the girders were increased by up to 146 percent and the induced fatigue deflections were reduced by up to 75 percent.

2.2 Previous Studies on Concrete Jackets

Concrete jacketing enhances both the strength and stiffness of the original member which is beneficial in case of seismic retrofitting and upgrading of the structural durability. El-Ebweini and Ziara [11] repaired six girders after corrosion by removing concrete cover and adding to the corroded part two longitudinal bars fixed with shear

dowels. The main differences in the specimens were in the type of the repairing material. The results conducted from this study were that the flexural capacity of the repaired girders was increased by 47% compared with the control girders.

Constantin E. Chalioris et al. [12] repaired five girders after shear failure by using self-compacting reinforced jacket. The results indicated that this rehabilitation method was a reliable one since the capacity of the repaired girders was fully restored according to the initial specimens.

Qasem Khalaf et al. [13] studied the flexural behavior of 26 reinforced concrete girders repaired using two techniques, concrete jackets and steel plates. The test variables were the aim of strengthening (shear or flexure), the technique used in strengthening (concrete jackets or steel plates) and the type of bond between the old concrete and the strengthening element (mechanical or chemical). The conducted results show that the specimens that strengthened by concrete jackets bonded either mechanically or chemically was more effective than that strengthen by steel plates.

Raval and Dave [14] tested the strengthening of reinforced concrete girders using concrete jackets. Ten girders were tested; four girders were prepared with smooth surface and other four with chipped surface while the remaining two girders were considered as control test girders. Four different techniques of bonding were used in this study. The results show that the girders with smooth surface and using jacket with combined shear dowels and bonding agent with micro-concrete was the most effective technique and for the girders with chipped surface, the most effective technique was using only micro-concrete and without use of shear dowels and bonding agent.

In this paper, an innovative technique is proposed to use the external prestressing method in a more easy-to-apply way for flexural strengthening of girders. This is done by means of embedding the draped profile tendons in a concrete jacket, externally bonded to the original girder. In the following sections, the experimental and analytical phases of the research are presented to verify the adequacy of the proposed technique.

III. EXPERIMENTAL WORK

The experimental phase of this research program consisted of two main stages, in the first stage two girders were cast with the cross-section shown in figure (1) and were loaded up to 80% of their ultimate moment capacity which was calculated by first principles.

In the second stage, the two tested girders of the first stage were strengthened and re-tested up to failure. In addition two girders untested girders were jacketed and then loaded

to failure. Figure (2) shows the cross-section and the cable profile of strengthened girders. The strengthened girders of stage two differs in the compressive strength of jacket and

the damage level in the original section of each girder. The properties of the girders in each stage are summarized in Table (1).

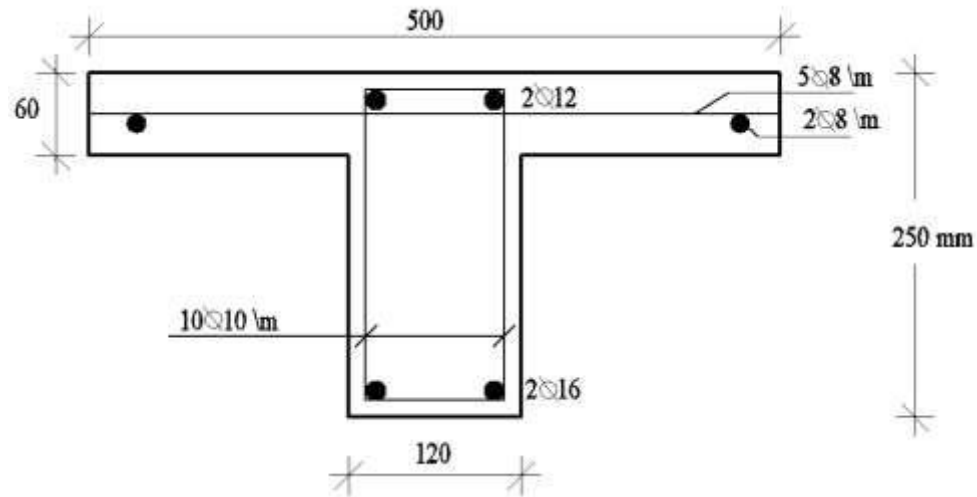
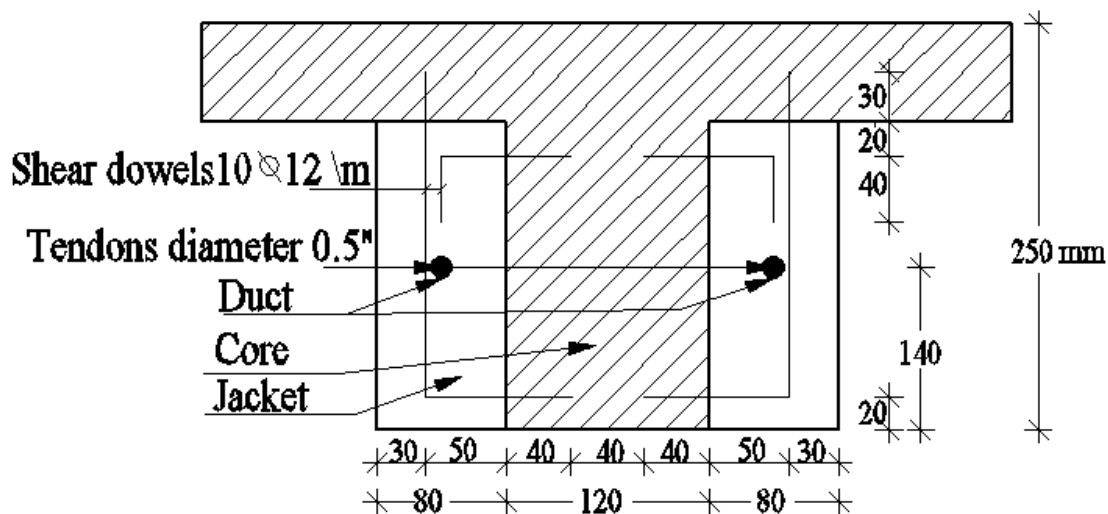


Fig.1: Cross-Section of all Girders in Stage 1



Cross-Section of Strengthened Girders in Stage 2

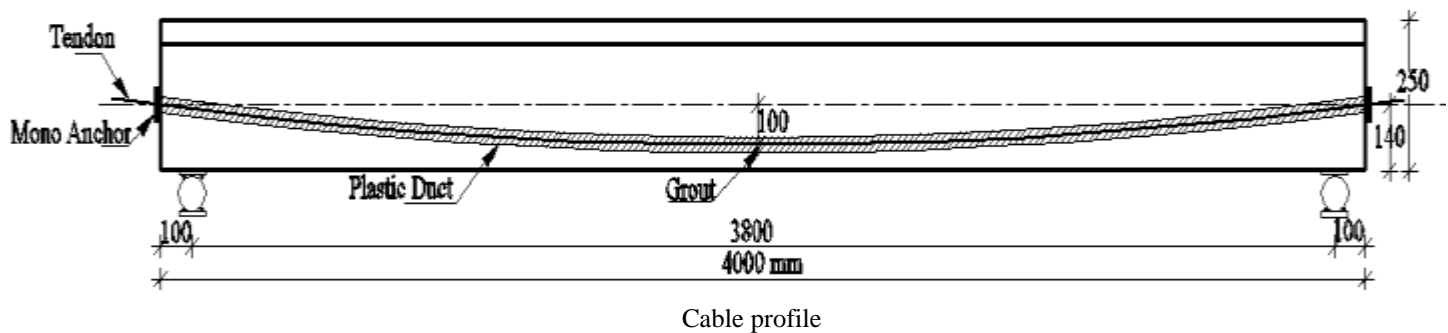


Fig.2: Cross-Section and Cable Profile of the Girders in Stage 2

Table.1: The main variables of girders in each stage

Stage No.	Specimen Label	Core f_{cu} (MPa)	Jacket f_{cu} (Mpa)	Damage level for core
1	RC 1-1	30	--	Loaded before strengthening up to 80 % of its loading capacity.
	RC 1-2			Loaded before strengthening up to 80 % of its loading capacity.
	RC 1-3			No loading for this girder in this phase.
	RC 1-4			No loading for this girder in this phase.
2	PC 2-1	30	30	80% before strengthening.
	PC 2-2		60	80% before strengthening.
	PC 2-3		30	0% before strengthening.
	PC 2-4		60	0% before strengthening.

3.1 Material Properties

The girders core for phase 1 was cast with a concrete having a 28-days cube compressive strength of 30 MPa. For phase 2 there are two different concrete mixes for the concrete jacket which as shown in table (1), the girder jackets' were cast with a concrete having a 28-days cube compressive strength of 30 and 60 MPa. Ordinary Portland cement and local natural sand and the coarse aggregate "crushed limestone" (Dolomite) with 10-mm maximum size were used. All the used materials were matched with the Egyptian Code of Practice (ECP 203) [15].

Silica fume was used to achieve concrete compressive strength 60 Mpa. The workability of the mix was improved by using a high-range water reduction admixture under a commercial name of Sika Viscocrete. The compressive strength of concrete was evaluated after 28-days of casting based on the cube (15.8 x 15.8 cm²), and was found to be as 30.94 and 61.63 MPa respectively.

Steel bars of grade 240/350 and grade 360/520 were used. The mild steel smooth bars of grade 240/350 which have minimum yield strength of 240 MPa and ultimate tensile strength of 350 MPa were used for bar size of 8mm. The high tensile steel deformed bars of grade 360/520 that have minimum proof strength of 360 MPa and ultimate tensile strength of 520 MPa were used for bar sizes of 12 and 16 mm. All the previous types of steel had constant modulus of elasticity of 210 GPa. High tensile, low relaxation PT tendons (ASTM A416, grade 270) [16] were used with a diameter 0.5" (12.7 mm).

3.2 Test Setup:

The experimental work was conducted in the reinforced concrete laboratory of the faculty of engineering at Ain Shams University, Figure (3) shows the test setup used in this study. All girders were tested as a simple span with a clear span of 3800 mm using two points loading system spaced 1200 mm.



Fig.3: Test Setup for all Girders in the Two Stages

3.3 Loading Protocol

Girders RC1-1 and RC1-2 were aligned before strengthening in test rig with the required effective spans and were subjected to an incremental loading till 80% (45 KN) and then strengthened with the jacket and retested till failure. While girders PC 2-3 and PC 2-4 were strengthened firstly and then tested to failure.

3.4 Instrumentation

The accuracy of the measurement devices determines the reliability of the experimental program results. In this

respect, a powerful combination of measurement devices was used to monitor and record the test outputs. Figure (4) indicates three 0.01mm accuracy Linear Variable Displacement Transducers (LVDTs) were used to measure the deflections at different points. The strain on the ordinary steel was measured by four electrical strain gauges (ES), mounted on the mid span for the top and bottom steel bars. All the LVDTs and electrical strain gauges of stage one and two were connected to the appropriate number of channel boxes (4 channels each). Finally, all the data were recorded by the data acquisition device.

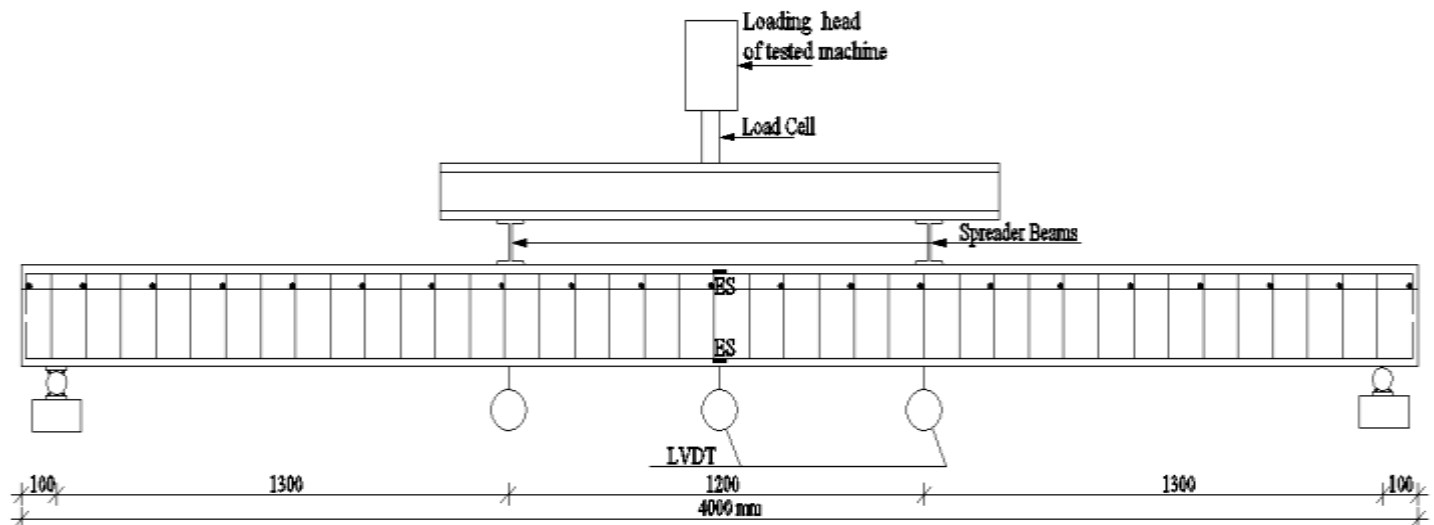


Fig.4: Deflection and Steel Strain Measurements for all Girders

3.5 Results and Discussion

3.5.1 Crack Pattern

In the first stage, the crack pattern of the two tested girders (RC 1-1, RC 1-2) was similar as shown in figure (5). First crack appeared in the constant moment zone at a load 15 and 15 kN for girders RC (1-1) and RC (1-2) respectively. With increasing loads till 45 kN, the number and width of the flexural cracks increased and propagated vertically towards the girders' flange and gradually covering longer portions of the girders' spans. About 25 cracks were observed in these two girders and the average spacing between cracks was about 8 cm.

In the second stage, the crack pattern of all the girders was similar as shown in figure (6). At the early stages of loading, first crack appeared in the constant moment zone at a load 60, 50, 50 and 50kN for girders PC 2-1, PC 2-2, PC 2-3 and PC 2-4 respectively. With increasing loads, flexural cracks increased in number and width while propagating

vertically towards the girders' flange axis and gradually covering longer portions of the girders. About 15 cracks were observed in these four girders, which is less than the number of cracks in the girders before strengthening, and the average spacing between cracks was about 12 cm. During this phase the vertical load increased to 126.45, 138.83, 142.00 and 150.00 KN for girders PC 2-1, PC 2-2, PC 2-3 and PC 2-4 respectively, which represent the failure load for each girder.

In summary, all girders failed in flexure through ductile failure as monitored for girders PC 2-1, PC 2-2, PC 2-3 and PC 2-4. All tested girders behaved the same mode of failure as crushing in the upper surface of concrete under one of the loading girders. Figure (7) shows mode of failure of each girder. It is noted that this crushing appeared after several flexural cracks have developed in the girders, hence the ductile nature of the failure.



Fig.5: Crack pattern of girders RC (1-1) and RC (1-2) respectively



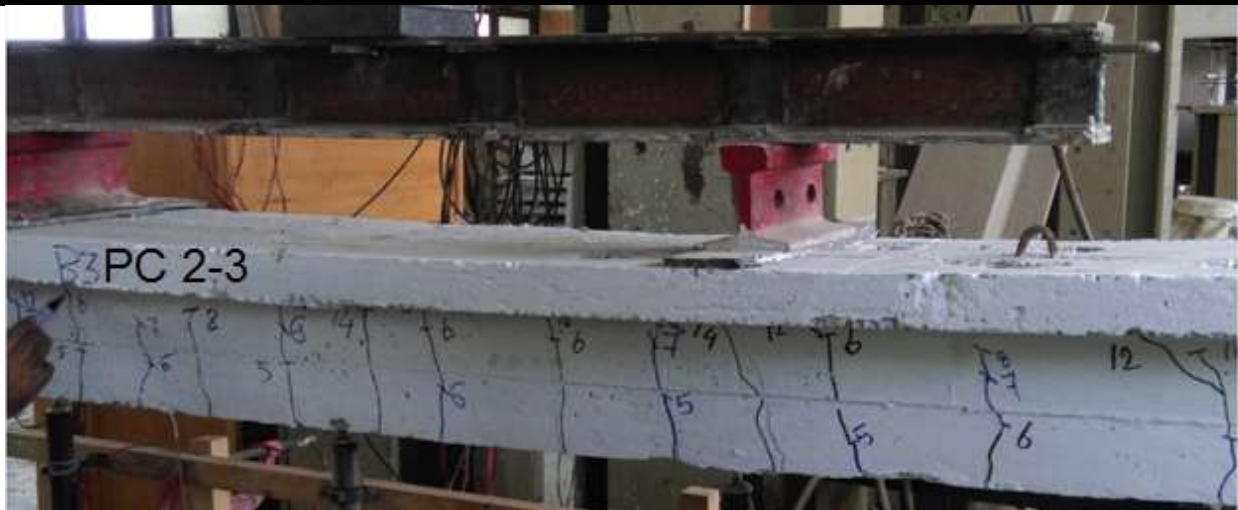


Fig.6: Crack pattern of girders (PC 2-1, PC 2-2, and PC 2-3 and PC 2-4 respectively)





Fig.7: Mode of failure of the girders (PC 2-1, PC 2-2, PC 2-3 and PC 2-4)

3.5.2 Load-Deflection Response

The vertical load and the mid-span deflection were measured for all girders in each stage. Figure (8) shows a comparison between the girders that were loaded on their original section firstly and then strengthened with the jacket and reloaded to failure. It was observed that the strengthened girders (PC 2-1 and PC 2-2) were stiffer than the girders (RC 1-1 and RC 1-2). At load 45 kN, the mid-span deflection [as shown in table (3)] of the girders PC 2-1

and PC 2-2 were less than the girders RC 1-1 and RC 1-2 by 47% and 63% respectively.

The flexural behavior for girders PC 2-1 and PC 2-2 were similar, but the load-carrying capacity of girder PC 2-2 was higher than girder PC 2-1 by 9.8% as shown in figure (9) which may be attributed to the difference between the jacket's concrete compressive strength for girders PC 2-1 and PC 2-2 as illustrated in the previous table (1).

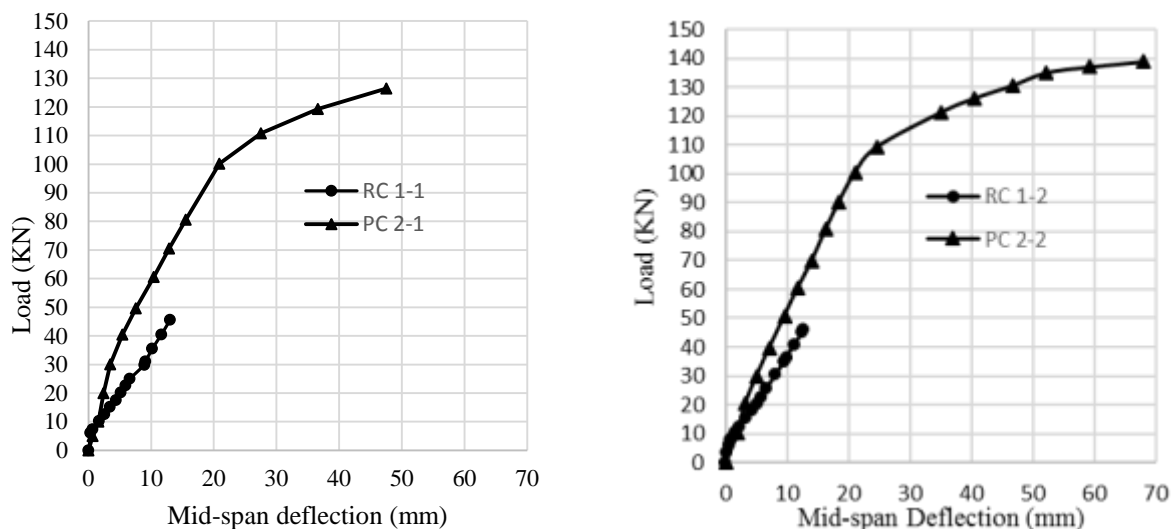


Fig.8: Load vs. Mid-Span Deflection

Table.3: Mid-Span Deflection at Load 40 kN

Girder Label	Mid-span Deflection (mm) at Load = 45 kN
RC 1-1	11.66
RC 1-2	11.13
PC 2-1	5.41
PC 2-2	7.01

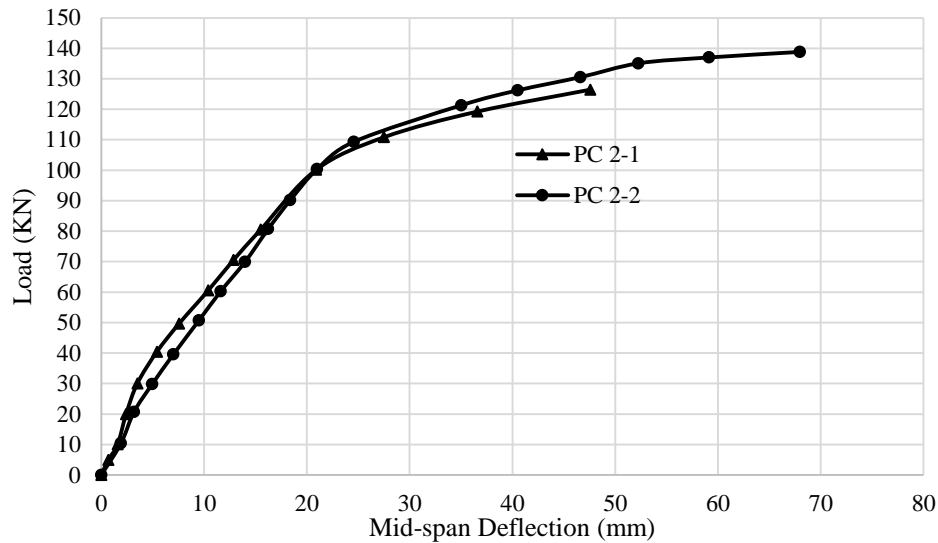
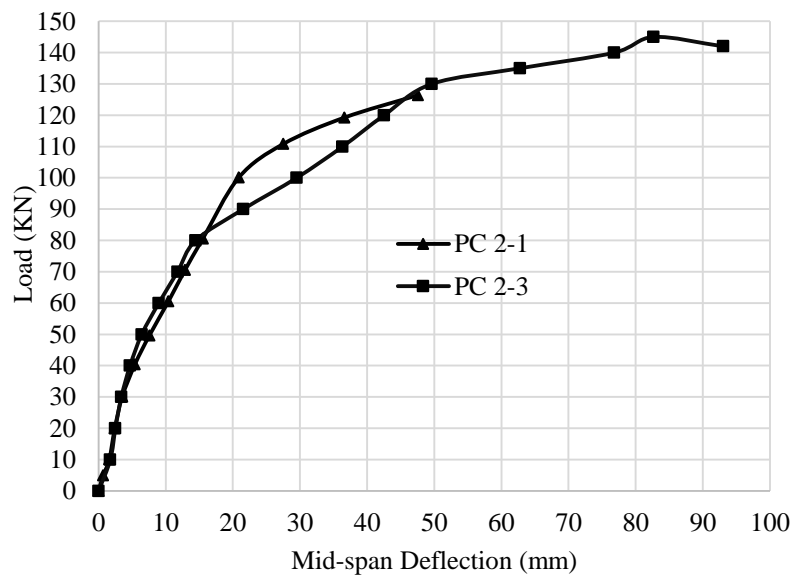


Fig.9: Load versus Mid-Span Deflection

Figure (10) shows a comparison between the strengthened girders (PC 2-1 and PC 2-3) and (PC 2-2 and PC 2-4), where the main difference between each two girders were the damage level of the original girder before strengthening as illustrated in the previous table (1). It was observed that the load-carrying capacity of the girder PC 2-3 and PC 2-4 were more than the girder PC 2-1 and PC 2-2 by 11% and 7.5% respectively.



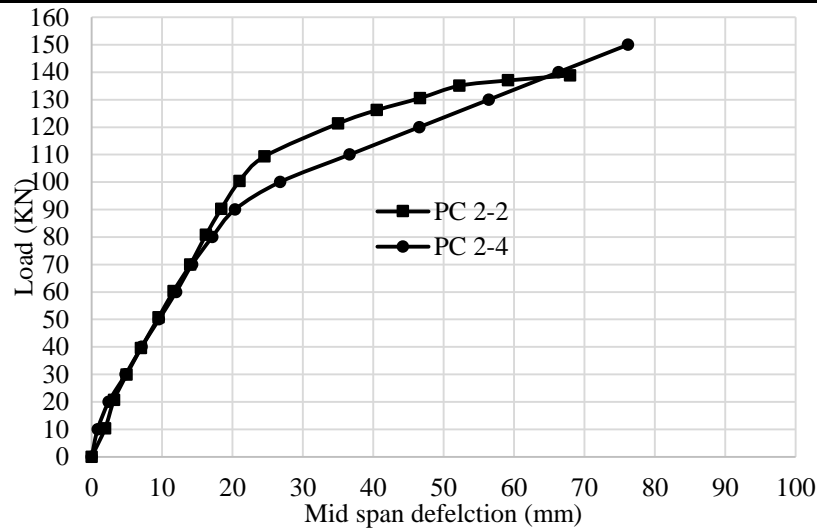


Fig.10: Load versus Mid-Span Deflection

The experimental results for the girders before and after strengthening were summarized in Table (2). Results were expressed in terms of their first-crack load, ultimate load-carrying capacity and ultimate displacement.

Table.2: Summary of the experimental test results

Phase no.	Girder Label	P_{cr} (KN)	Δ_{cr} (mm)	P_u (KN)	Δ_u (mm)
1	RC 1-1	15	3.43	--	--
	RC 1-2	15	3.23	--	--
2	PC 2-1	60	10.41	126.45	47.57
	PC 2-2	50	9.50	138.83	67.69
	PC 2-3	50	6.42	142.00	93.03
	PC 2-4	50	9.56	150.00	76.20

P_{cr} = first crack load, P_u = ultimate Load, Δ_{cr} = mid-span deflection at first crack, Δ_u = mid-span deflection at failure load

IV. FINITE ELEMENT ANALYSIS

Modelling using finite element approach for reinforced concrete is a delicate task. Both elastic and plastic behavior of concrete in tension and compression is to be incorporated while creating a proper model. The behavior of reinforced concrete under tension can be incorporated using tension stiffening. In this study, the ABAQUS software package which uses the finite element method was used to model the flexural behavior of the girders in stages one and two. Concrete damaged plasticity material model was implemented for the concrete continuum.

4.1 Material Properties for the Numerical Simulation

4.1.1 Concrete

The concrete damaged plasticity (Cdp) model depends on assuming that the failure mechanism will be as the compressive crushing and tensile cracking. The uniaxial compressive strength of concrete core in stage one was 30 MPa and for stage two the compressive strength of post-

tensioned concrete jacket was 30 and 60 MPa. The plastic strain of concrete is considered as 0.035 and which was used in the analysis. The Poisson's ratio of concrete is suggested as $\nu = 0.2$.

4.1.2 Steel Reinforcement

The steel rebars were modeled as truss elements with yield stress 360 MPa and 240 MPa for the main steel reinforcement and for the stirrups respectively. The tendons used in phase two for post-tensioned concrete jackets were modeled as truss elements with yield stress 1674 MPa. The young's modulus of steel reinforcement and tendons was 200 GPa and poisson's ratio 0.3. The rebars and tendons can be defined as a one- dimensional strain element and is embedded in the concrete. This can be achieved by using embedded constraint criteria in ABAQUS.

4.2 The Finite Element Mesh

In order to get accuracy in results, all the elements of FE model were assigned the same mesh size so that each two

different materials can share the same node among them. The mesh type selected in the model is given below. The mesh elements for concrete core and post-tensioned concrete jackets were taken as 3D solid element which is called C3D8 and for steel reinforcement and tendons 2D truss element is assigned which is called T3D2.

V. DISCUSSION OF ANALYTICAL RESULTS

5.1 Load–Displacement Response

The mid-span deflection was calculated for the bottom face of the girders from the numerical model. Figures (13) and (14) show the load-deflection curves of the girders before and after strengthening for both the tested beams and the numerical simulation results. The results from the finite element correlate well with those from the experimental

data at both stages. From these figures, it can be concluded that the variation between the experimental and analytical results ranged from 6% to 10% in terms of maximum load and peak deflection response. The overall shape of the load-deflection relationship matches to an acceptable degree.

It can be clearly noted from the figures that the model is quite capable of capturing the variations in the stiffness along the elastic and inelastic domains of the response. Initial stiffer response is observed for the analytical simulation results which may be attributed to two reasons, firstly the nature of the finite element simulation which mathematically should yield a stiffer response and secondly the minor sliding between the jacket and the base concrete which is not represented in the analytical model.

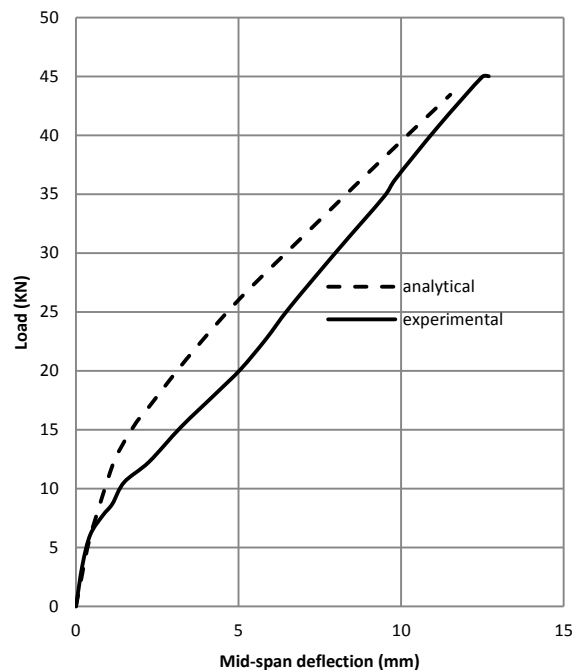
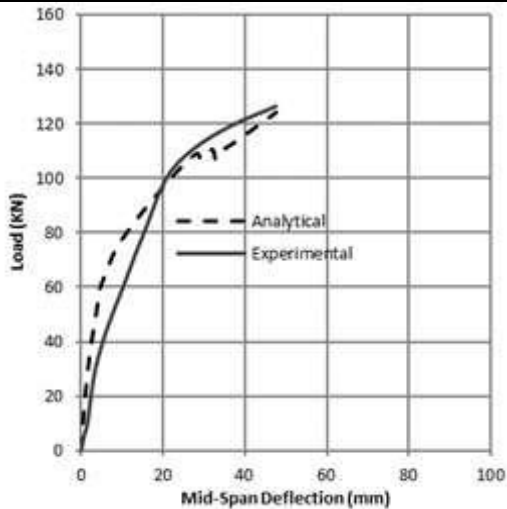
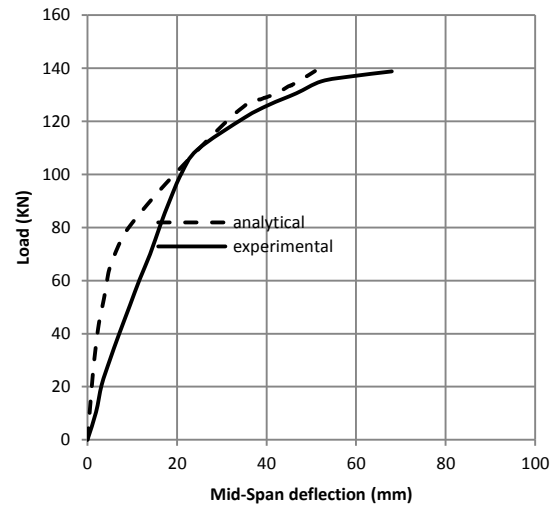


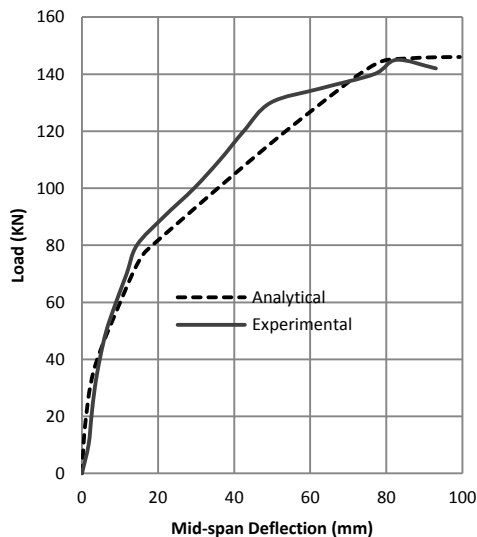
Fig.13: Load-Mid span deflection before strengthening



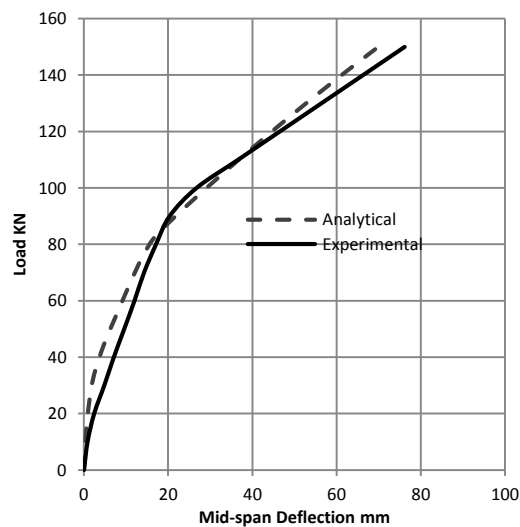
a: Load-Mid span deflection for (PC 2-1)



b: Load-Mid span deflection for (PC 2-2)



c: Load-Mid span deflection for (PC 2-3)



d: Load-Mid span deflection for (PC 2-4)

Fig.14: Load-Mid span deflection for analytical and experimental results of girders PC (2-1), PC (2-2), PC (2-3) and PC (2-4) after strengthening

VI. CONCLUSIONS

This paper presented an experimental and analytical investigation in the flexural response of flanged RC girders strengthened with post-tensioned concrete jackets. The results of the experimental and analytical investigation yielded the following conclusions.

Post-tensioned concrete jackets can be used as an effective strengthening method for damaged and undamaged girders. Both the strength and the stiffness of the girders were enhanced significantly. This means that this technique can be used for mitigating the effect of increase deflections as well as loss of strength.

The stage at which the jacket is introduced slightly affects the strength and stiffness of the girders. The difference is

not large enough to be considerable. This clearly demonstrates one of the merits of this system which is that it can be used on damage substrate without initial repair.

Increasing the jacket concrete compressive strength may result in a slight increase in the strength by about 10% of the original girder capacity.

A numerical model based on the finite element method was developed and compared to the experimental results. The results show a very good match and correlation between the two results. This model will be used in the future to develop a design methodology for the post-tensioned jacketing technique.

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